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HIGHWAY 400 RUTTING STUDY: - AT RUTHERFORD RD. AND AT HIGHWAY 9

MATERIALS INFORMATION

ENGINEERING MATERIALS OFFICE

HIGHWAY 400 RUTTING STUDY:
- AT RUTHERFORD RD.
AND AT HIGHWAY 9

by

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HIGHWAY 400 RUTTING STUDY: - AT RUTHERFORD RD. AND AT HIGHWAY 9

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Abstract:

In order to alleviate some of the problems associated with the rutting on Highway 400 between Toronto and the Holland Marsh, Contract 90-48 called for selected resurfacing of the driving lane in various locations. Two cross-sectional slab samples that were representative of the rutting were removed from the driving lane as part of this contract. These slabs were examined and put through a series of laboratory tests in order to characterize the materials used, and to determine the cause of rutting on that section of Highway 400.

This report examines the construction history of the sample locations and their present environment, including traffic conditions and summer temperatures. It describes the experimental program that the recovered samples were subjected to and the results subsequently obtained. Conclusions and recommendations regarding the rutting problem on Highway 400 are made, and improvements to future investigations of this type are suggested.

Abrégé:

Afin d' éliminer certains des problèmes associés à l'orniérage de l'autoroute 400 entre Toronto et le Holland Marsh, le contrat 90-48 prévoyait la réfection du revêtement de la voie de droite en divers endroits. Dans le cadre de ce contrat, on a prélevé deux échantillons transversaux qui étaient représentatifs de l'orniérage de la voie de droite. Ces échantillons ont été inspecté et soumis à des tests de laboratoire visant á déterminer les caractéristiques des matériaux utilisés et à établir la cause de l'orniérage de ce tronçon de l'autoroute 400.

Le présent rapport examine la construction de la route aux endroits choisis et l'environnement actuel, y compris les conditions de circulation et les températures estivales. Il décrit le programme expérimental auquel les échantillons ont été soumis et les résultats obtenus. Le rapport tire des conclusions, fait des recommandations sur le problème de l'orniérage de l'autoroute 400 et suggère des améliorations à apporter à ce genre d'études.

Keywords: Rutting profile, pavement slab sampling, asphaltic concrete, material characterization, mix gradation, laboratory testing, gyratory testing machine, wheel-tracking machine.

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1.0 INTRODUCTION

Rutting has become a problem on various high volume freeways and highways in Ontario, one such area being Highway 400, between Toronto and Barrie. As part of the Ministry of Transportation of Ontario's (MTO) ongoing program to alleviate rutting and other highway distresses, Contract 90-48 called for selected rehabilitation of Highway 400 between Highway 7 and Canal Road at the Holland Marsh at a cost of $1 400 000. Included in the work for Contract 90-48 was a special provision for the removal of two cross-sectional slab samples of the existing rutted pavement. These samples were examined and characterized by the MTO's Bituminous Evaluation Laboratory with regard to their material and mixture properties to pinpoint the causes of rutting on that particular portion of Highway 400.

2.0 PURPOSE AND SCOPE

This pavement investigation was conducted with the following purposes in mind:

a) to determine the causes of rutting in the section of road represented by the recovered samples through an analysis of the individual components as well as the asphalt mixtures,

b) to disseminate information on the findings so that an appropriate monitoring and associated maintenance plan for this section of highway could be devised,

c) to acquire data for the establishment of a database for reference in future rutting investigations.

The scope of this investigation was to act as a "first step" in a series of pavement investigations. It was used to explore additional ways of evaluating rutting, to smooth out problems encountered within testing procedures, and to determine the most efficient methods of conducting future investigations to suit both Regional and Head Office needs.

3.0 BACKGROUND

3.1 Sample Location and Description

Pavement samples representative of the rutting on Highway 400 were obtained as part of the work of Contract 90-48. They were cut from the truck lane at two locations. The first sample was taken from the northbound side of the highway, approximately 13 km north of Highway 401. It was located at station 3+800, about 300 m south of the Rutherford Road overpass, on an uphill grade of 2%. The second sample was taken from the southbound side, approximately 37 km north of Highway 401. This sample was located at station 26+900, approximately 60 m north of the Highway 9 overpass, on an uphill grade of 2.5%. For the purpose of this report, these two samples will be referred to as the "northbound sample" and the "southbound sample" respectively.

Each sample was cut from the pavement 1200 mm (4 ft) along the direction of travel and full width across the existing 3.65 m (12 ft) truck lane. The sample was further sawn to produce two transverse strips each 600 mm (2 ft) in length and 3.65 m (12 ft) wide. These strips were cut into 24 blocks, each 300 mm (1 ft) wide. The northbound sample

was recovered to the underlying concrete base, while the southbound sample was recovered to the underlying granular base. The depth of each slab was nominally 400 mm (1 ft 4 in).

3.2 Traffic Volumes and Patterns

Highway 400, once referred to as the Toronto-Barrie Highway, was constructed as a two lane divided highway in the late 1940's and early 1950's. Since its full opening in 1952, the number of vehicles using it has increased dramatically. The annual average daily traffic, (AADT), has increased approximately seven fold since 1955, with growth averaging over 25% in each five year period since that time. In 1988 Highway 400 carried a two-way AADT of 38 000 vehicles per day at Highway 9, and nearly twice that number, 75 700 vehicles per day, at Rutherford Road. Figure 1 shows the growth in AADT at each sample location in five year intervals since 1955 (1).

The type of traffic using Highway 400 shifts from commuter to recreation as you proceed northward from Toronto to Barrie. According to the MTO's Transportation Capital Branch analysts, the Rutherford Road area is classified as "commuter recreation" type traffic, while the Highway 9 area is referred to as "intermediate recreation". Because of this presence of recreational traffic, there is a seasonal variation in the number of vehicles using Highway 400, evident by the summer average daily traffic (SADT) being typically 34% greater than the AADT.

During 1988 a commercial vehicle survey was conducted by the Provincial Transportation Systems Office of the MTO. Truck counts were taken once during each of the four seasons of the year, and included data gathered from two Highway 400 weigh stations. The first of these was the King station, located in the northbound lane, 2.1 km north of the King Side Road. The second was the Woodbridge station, located 3.7 km north of Highway 7 in the southbound lane. Because of their proximity to the sampling locations, the Woodbridge truck count data was applied to the northbound sample while the King station data was applied to the southbound sample. Information from these stations provides a very good estimation of the truck traffic at the respective sites.

The four seasonal truck counts showed that just over 2300 trucks per day were travelling over the section of roadway where the southbound sample was taken. This corresponds to 12.1% of the one way AADT in that area. The count at the Woodbridge station showed a seasonal average of just under 2900 trucks per day passing over the northbound sample. This corresponds to 7.6% of the respective one way AADT. The counts show that there are more trucks on the northbound pavement than on the southbound, even though the northbound percentage is smaller. This is because of the higher AADT for the northbound sampling location and that pavement's use by more commercial and private vehicles from the large industrial centre of Metro Toronto nearby.

Two types of truck configurations make up more than 81% of the total truck traffic on Highway 400. These are the 4 axle and 5 axle tractor-trailer combinations. They carry loads that range from 5000 kg per axle group to 14 000 kg per axle group on average, with total vehicle weights of approximately 45 000 kg.

3.3 Summer Temperatures

Like all Ontario roads, Highway 400 is subjected to cold temperatures in the winter months and warm temperatures in the summer months. The annual temperature range experienced by Highway 400 south of Barrie is typically -30°C to +30°C. While pavements must be designed so that they are flexible enough to withstand low temperature

cracking induced by extreme winter conditions, design must also take into account the high temperatures that are present in the summer months which are a significant factor in the rutting of asphalt pavements.

Data from nearby Environment Canada climatological stations was used to estimate the ambient air temperature conditions present in the vicinity of each sampling location. The station located at Woodbridge was taken to represent the northbound sample, while Bradford stations were used to represent the southbound sample conditions. Figure 2 presents the annual mean temperature, as well as the "summer mean" and "summer maximum" temperatures since 1971, the year of the most recent construction. Here, "summer" refers to the three months of June, July, and August only. The summer mean temperature plotted in Figure 2 refers to the average of the monthly means of June, July, and August for each year, while the summer maximum temperature line represents the single highest temperature recorded during those same three months. Since 1971, the general areas from which these two samples were recovered have experienced peak temperatures of between 30°C and 39°C during those three summer months (2,3,4).

Although isolated peak temperatures could influence rutting, it is the prolonged periods of high temperatures that are of particular importance. Table 1 gives a breakdown of high temperatures in each sample area for June, July, and August of 1989 and 1990. For the three summer months of those two years, the area represented by the northbound sample had an average of 20 days (22% of the summer days) with an extreme temperature reading of 30°C or more, and an average of 68 days (74%) with an extreme temperature of 25°C or more. Also in that period there were on average, 18 occurrences of three consecutive days with temperature readings of 25°C or more. Over the same two year period, the area of the southbound sample had an average of 12 days (13%) with an extreme temperature reading of 30°C or more, and 58 days (63%) of 25°C or more. There was an average of 15 occurrences of three consecutive days with a reading of 25°C or more.

3.4 Construction History

In the contract drawings (Figures 3a and 3b) of highway construction on the two sites prior to 90-48, it was noted that HL5 was to be used for the binder mixes. But the Bituminous Construction Reports revealed that HL6 mixes were actually placed. Since these reports contained the largest source of mix design information, they were therefore considered to be correct and this report refers to all binder lifts as HL6.

(i) Northbound Highway 400 at Rutherford Road

There were four capital construction contracts prior to Contract 90-48 encompassing the area from which the northbound sample was taken. While no records exist for the first two contracts (Contracts 46-10 and 50-180) on this section of roadway, contract drawings and Bituminous Construction Reports were available for the later two contracts, i.e., Contract 58-291 and Contract 71-14.

Although cross-sections from the 58-291 contract drawings are not included in this report, they indicated that the existing two lanes of portland concrete pavement placed in 1950 were resurfaced with a bituminous pavement system consisting of an HL6 padding course where required, two 50 mm (2 in) HL6 base courses, and a 32 mm (1.25 in) HL1 surface course. This work was carried out and completed in the summer of 1958, and was eventually resurfaced under Contract 71-14 in 1972.

Contract 71-14 called for widening each of the existing two lanes from 3.3 m (11 ft) to 3.65 m (12 ft), as well as construction of a third lane. This was accomplished by shifting the existing crown to the right by 0.9 m (3 ft) and adding an inside widening of 3.0 m (10 ft) on the left or median side, and an outside widening of 1.2 m (4 ft) to the right of the existing roadway. A 1.2 m (4 ft) wide strip of steel wire mesh was placed on each longitudinal widening joint. For the outside widening where the slab sample was taken, the contract drawings call for 200 mm (8 in) of HL6 base plus a 50 mm (2 in) lift of HL6 placed flush with the existing pavement. An HL6 padding course was placed on the old truck lane, mainly to facilitate the crown shift. Then the whole width was overlaid by a 45 mm (1.75 in) HL6 binder course followed by a 32 mm (1.25 in) HL1 surface course. A typical section taken from the contract drawings is shown in Figure 3a. This work was carried out and completed in the summer of 1972.

(ii) Southbound Highway 400 at Highway 9

The construction history of the southbound sample was very similar to that of the northbound sample. There were three previous construction contracts on that particular section of Highway 400: Contract 46-13, Contract 51-33, and Contract 71-12. Again, only a limited amount of information was available for the first two of these contracts. In fact there are no records for Contract 46-13, while only a brief mention of the aggregates and asphalt cement (AC) used in the hot mix paving of Contract 51-33 was found in the Bituminous Section files. However, contract drawings and a Bituminous Construction Report exist for Contract 71-12.

The three lifts of hot mix laid under Contract 51-33 (approximately 130 mm or 5 inches) were placed directly over the sub-base. However, the 71-12 contract drawings indicate a cross-section that is very similar to that previously described for Contract 71-14. This typical cross-section is shown in Figure 3b. Like 71-14, Contract 71-12 also called for widening from two lanes to three by shifting the crown and adding an inside and outside widening. This outside widening again consisted of 1.2 m (4 ft) extending to a depth of 250 mm (10 in) below the top of the old surface course. A padding course was then placed over the original roadway, extending over the new widening as well, to accomodate the crown shift. This was followed by 38 mm (1.5 in) of HL6 binder, then 32 mm (1.25 in) of HL1 surface course. Unlike Contract 71-14, steel mesh was not placed on the joint between the original roadway and the new widening.

Contract 71-12 took three construction seasons to fully complete. The Bituminous Construction Report indicates padding and binder courses were placed from July to October 1971 then completed in June 1972, while the major portion of the surface course was placed in October 1971 and June 1972. The report also shows that settlement problems were encountered through Holland Marsh causing delay of surface course placement in this area until 1973. Our sample location lies south of this problem area, so we can assume that paving in the area of the sample was completed in the summer of 1972.

3.5 Mix Design History

Information concerning the mix designs of Contracts 58-291, 71-14, and 71-12 were obtained from Bituminous Construction Reports kept on file in the Bituminous Section. These reports shed some light on the types of aggregates and asphalt cements used, but did not characterize every lift entirely. This was due to the fact that the mix designs of the HL6 binders changed during construction. Two different fine aggregate sources and AC contents were used for the HL6 binder courses placed under Contract 71-14, while two aggregate sources with three different gradations and three different AC contents were used for the HL6 binder courses of Contract 71-12. Tables 2a and 2b

summarize the mix designs relevant to the northbound and southbound samples respectively. Also included in each table is a description of which mix design applies to the respective lifts of each cross-section, based upon information gathered from various test results and experience.

4.0 TESTING PROGRAM AND RESULTS

4.1 Observations of Field Samples

After shipment from the field to the evaluation laboratory, the individual blocks of each slab were reassembled to provide a reconstructed view of the pavement cross-section. Each block was labelled from left to right for future reference, and the lifts were numbered from the 7th, or top lift, downwards to the 1st, or bottom lift. Figures 4a and 4b illustrate the reconstructed profile of the northbound and southbound samples respectively.

Upon visual examination of the slab samples and review of the design cross-sections, it became apparent that each sample contained an additional lift besides those which were expected. In both cases this extra lift was located directly above the original surface course (lift 3), easily distinguishable due to the green traprock present in the HL1 mix. Instead of just two binder lifts placed over this old HL1, there were actually three lifts present. In the case of the northbound sample, the mixes of these three lifts looked very similar. The aggregates were fairly consistent in appearance, and the AC contents also appeared to be similar. Material evaluation testing later confirmed this similarity in the northbound sample.

According to the northbound design cross-section, a 4 foot wide section of wire mesh was to be placed above the old HL1 to bridge the existing pavement with the shoulder-side widening. Bituminous Construction Reports also mention that the wire mesh was insufficiently covered by the padding. Based upon these observations, it was concluded that the extra lift in this sample was simply more padding placed at the time of construction to correct the mesh problem and yet maintain crossfall.

However, the fourth lift of the southbound sample appeared to be substantially different from the two lifts directly above it. It looked to be a darker, richer mix, with a finer graded coarse aggregate and a higher AC content when compared to other binder courses present in either slab. The extra lift in this cross-section is most likely a maintenance patch, an additional lift of asphalt placed by the MTO Maintenance Section on an interim basis to temporarily repair any problems on the driving surface until the next capital construction contract provided complete rehabilitation. The fact that this lift acted as a driving surface for some length of time was confirmed by the presence of a yellow stripe located on top of the fourth binder of block D-9 (see Figure 4b). The location of this block corresponds to the edge of the old two lane driving surface, where the stripe would have been marking the edge of pavement. An analysis by the Chemical laboratory confirmed that it was paint, and that it was placed prior to 1971, i.e., at least one year before Contract 71-12.

Another point of interest was the variation in the structural geometry of the widening portion of the two samples. The northbound sample exhibited extra depth under the wire mesh on the right side of the slab corresponding to the pavement widening, as expected considering the design cross-section. However, the widening portion of the southbound sample was not as deep as the main sub-base section, contrary to the 71-12 contract drawings. Instead of having two lifts of 100 mm (4 in) each plus a 50 mm (2-in) lift flush with the old surface course, there were four lifts, each of which was only

approximately 25 mm (1 in) in depth. This resulted in a sub-base of approximately 100-mm (4 in), rather than 250 mm (10 in). Although it is possible that the extra depth was not recovered from the field, this scenario is not likely since a visual examination indicated that the bottom lift of the widening was placed on a granular sub-base.

Longitudinal cracking was observed on blocks from the widening portion of the southbound slab, occurring through the full length of the left underside of blocks C-11 and D-11, and through the full length on the surface of the right-most blocks C-12 and D-12. This cracking pattern is shown in Figure 4b. The pattern of these (tension) cracks suggests that the pavement was subjected to some degree of structural failure. This failure is believed to have contributed in part to the right wheel track rutting on that particular section of roadway.

It appears from the cross-sectional view of the southbound profiles (Figure 4b) that the top two layers, 6 and 7, experienced the greatest rutting. This was indicated by the slight increase in thickness on the right side of the left wheel track for layer 7, and on the left side of the left wheel track in lift 6 with slightly more in the binder. It appears from the profiles in Figure 4a that rutting also occurred in the northbound upper binder mixes (lifts 6 and 7). This distortion in the upper binders supports findings by other researchers (5) that maximum horizontal shear stresses can occur at depths between 50 to 100 mm.

4.2 Laboratory Testing

In order to perform laboratory testing on material from individual lifts, the recovered blocks of each sample were broken down by the use of a specially made heating apparatus, consisting of a plywood box fitted with a heater and circulating fan. Each block was placed into this apparatus in such a way that allowed only the bottom lift to be softened by the heat and broken off at one time, thereby exposing the next lift, which was then heated and broken off in sequence. This process was repeated for 9 of the 12 blocks of each slab. The remaining three blocks were sawn or cored to required sizes for other testing.

4.2.1 Material Evaluation

The following tests were performed:

Mix	Aggregate	Recovered AC
-extraction and gradation -BRD and MRD -% AC	-mineralogy -% crushed	-viscosity -penetration

(i) Extractions

The recovered gradations of the mixes were plotted in Figures 5a to 5c. Furthermore, the AC content, percent passing the 4.75 mm sieve and percent passing the 75 μm sieve were compared, on lifts that this information was available for, to mix design values and to the average of mix check results performed by the Regional laboratory at the time of construction (Table 3).

The relationship between the observed HL6 lifts and their respective mix designs was established based upon the AC contents and gradations from these extraction results, estimates of the construction schedule, and bulk relative density testing described later. In

the case of the northbound sample, lifts 4 and 5 correspond to mix design 1, while lift 6 corresponds to mix design 2, as described in Table 2a. In the case of the southbound sample, mix design 2 applies to lift 6, while mix design 1 applies to lift 5, as described in Table 2b. As outlined earlier, the 4th lift is believed to be a maintenance patch, with an unknown mix design and no construction records.

(a) Northbound Samples:

Examination of the gradations from the extraction results of the northbound sample indicates many of the mixes are very fine (Figures 5a and 5c) with the top five lifts near the maximum limits in use during that period (6), as summarized below. This fineness is more apparent at the present time where HL1 and HL6 are currently allowed a maximum limit of 5 and 6% on the 75 μm sieve respectively (7).

Sieve Size	75 μm		150 μm	300 μm	4.75 mm
Mix Type	HL1	HL6	HL6	HL6	HL6
Maximum % Passing*	7	8	10	22	55
% Passing (lift#)	5.5 (7)	6.7 (6)	11 (6)	25 (5)	62.3 (5)
	6.2 (3)	7.1 (5)	12 (5)	24 (4)	56.2 (4)
		5.9 (4)	11 (4)		

* = *reference (6)*

Mix checks performed at the time of construction confirmed the fine gradings, as evidenced by the pass 75 μm results. In fact, the checks indicate higher dirt contents than those from the extractions done in this study or the mix designs, i.e.:

	% pass 75 μm		
	HL6 - (lifts 4,5)	HL6 - (lift 6)	HL1 - (lift 7)
mix check	8.6	8.4	6.4
mix design	4.4	3.0	3.2

Although mix check gradations were not available for lift 3 northbound from Contract 58-291, our results show that it contains more than twice as much (6.2%) pass 75 μm material as the original mix design had called for (2.4%).

In examining the extraction results from this study it is apparent that the AC content of lifts 7, 3, and 2 from the northbound sample are at or below the allowable ±0.3% tolerance from mix design. In comparison, the Contract 71-14 mix checks indicated that the AC contents of the lifts of the northbound sample were within the tolerance of ±0.3% of the mix design value with the exception being the 6th lift, which was higher than the target by 0.4%.

(b) Southbound Samples:

The problem of excessive pass 75 μm material is not nearly as severe as for the northbound sample. As shown below, only two of the mixes which originally met specifications on the 75 μm sieve size are marginally finer by current specifications.

Sieve Size	75 µm		4.75 mm
Mix Type	HL1	HL6	HL6
Maximum % Passing*	7	8	55
% Passing (lift#)	6.0 (3)	6.5 (4)	59.3 (6)

* = *reference (6)*

Mixes from lifts 7, 6, and 5 are within specification, but have approximately twice the amount of pass 75 µm material specified in the mix design, ranging from 4.3 to 4.9% as shown below.

	% pass 75 µm		
	HL1 - (lift 7)	HL6 - (lift 6)	HL6 - (lift 5)
recompacted mix	4.8	4.3	4.9
mix design	2.0	2.0	2.6

As the mix design history indicated that only one Contract, 71-12, was placed after Contract 51-33, it was expected that lift 4 would be an HL6 levelling binder. After examination, it was apparent that lift 4 did not fit the HL6 gradation. Instead, it resembles an HL3 mix more, when plotted against the HL3 grading envelope of Figure 5c; as such, it is off-specification only on the 75 µm and 150 µm sieves, and slightly on the 4.75 mm sieve. This difference in gradation between lift 4 and lifts 5 and 6 contributes to the belief that lift 4 was a maintenance mix placed some time prior to Contract 71-12. However, for discussion purposes in this report, it will still be referred to as an 'HL6 upper binder'.

Based on the mix design data, the AC contents of the top three lifts of the southbound sample are within the tolerance but the construction mix checks indicate lifts 5, 6, and 7 to be higher by 0.4% to 0.5% AC.

(ii) Asphalt Cement Evaluation

The penetration of the recovered asphalt cement (AC) was determined at 4°C, 10°C, and 25°C, using MTO test procedure LS-200 (8). Kinematic and absolute viscosities of the recovered AC were evaluated using procedures described in LS-202 (8) and LS-206 (8) respectively. Table 4 displays the above values.

A plot of viscosity and penetration versus temperature for every lift using a Shell Bitumen Data Chart produces some variation in the penetration index, especially so because of lifts 2 and 3 northbound with the high absolute viscosity results.

The 25°C penetration values are perhaps a little high compared with what might be expected of an 85/100 grade AC from pavements of 20-40 years old. Penetration values for the various lifts from both samples averaged approximately 45 as compared to typical values of 55 to 60 obtained immediately after construction.

(iii) Coarse Aggregate Description and Percent Crushed

In (9) it was shown that the use of rounded aggregate in the binder lifts greatly contributed to the failure of the pavements examined at that time. To determine if the same problem applied to the pavements sampled in this project, the coarse fractions of the extracted aggregates from all binders of each slab sample were analyzed to determine the

amount of crushed coarse aggregates in those particular lifts. The coarse aggregates from lifts 7 and 3 of each slab were not analyzed since they originated from a 'traprock' source and were therefore assumed to be 100% crushed.

The percent crushed analyses were conducted in accordance with LS-607 (8) and LS-617 (8). These two tests determine the percentage of coarse aggregate particles having one or more faces crushed, and two or more faces crushed respectively. As well, the percentages of particles with three or more faces crushed and with all faces crushed, were determined on the 6th, 5th and 4th lifts of each slab sample. The LS Manual defines a crushed particle as a piece of coarse aggregate which has at least one well defined face resulting from fracture. Particles with smooth faces and rounded edges, or with only small chips removed, are not considered crushed. The results of the percent crushed testing are presented in Table 5.

Upon examination of these results, one immediately notices the large difference in the percentage of crushed/uncrushed coarse aggregates present in the upper binders (lifts 4, 5, and 6) of the two samples. The aggregates of the southbound sample exhibited at least 92% in each of the percent crushed tests and contained only 8% of uncrushed particles. The aggregates from the same lifts northbound which contained 25% of uncrushed material, dropped from 74% for one or more faces crushed to under 30% for all faces crushed.

A description of each aggregate's mineral composition was summarized from a partial petrographic analysis of the extracted aggregates from each lift, with individual minerals being included in the description if they formed at least 5% of the sample. This summary is presented in Table 5 along with the percent crushed data.

The aggregate portion retained on the 4.75 mm sieve for the surface courses (lift 7 in both directions) consisted of a blend of traprock, granite, and volcanic coarse aggregate. An unexpected finding was the difference in percentage of carbonitic material in the binders (4th, 5th, and 6th lifts) for both the northbound and southbound samples. The percentage of carbonates varied between 49% and 73% for the northbound, whereas the southbound coarse portions were entirely carbonates. Considering the high percentage of crushed material and the carbonitic composition of the southbound upper binder coarse aggregate, one can be sure this is a quarried material. On the other hand, the coarse aggregates of the upper binders of the northbound sample are most likely from a gravel source, because of their lower percentage of crushed material and jumbled composition.

4.2.2 Mix Characterization

The slab samples provided an opportunity to examine the different properties under recompacted, and undisturbed or 'as sampled' conditions for the same mixture. This was done by making beams and briquettes from the slab sample mixes and testing these by the Marshall method, the Wheel Tracking Machine and the Gyratory Testing Machine at various testing configurations.

(i) Marshall Properties of Recompacted Mixes

The recompacted Marshall characteristics of BRD, MRD, voids, stability, and flow were determined for the mixes of each lift, with material taken from blocks representing the wheel track area and from blocks located between the wheel tracks. Standard procedures found in LS-261 through LS-265 were followed (8). The voids, stability, and flow values were compared to mix design values and to mix check data obtained at the time of construction, where this information was available. The recompacted VMA calculations

were not made since the specific gravities for the individual coarse and fine aggregates were unknown. However, the VMA values from the mix checks were examined and compared to mix design values, where possible. This comparison is presented in Tables 3a and 3b, along with AC content and percent passing 4.75 and 75 µm sieves (see also Figure 5). Also, the field density of each lift was measured following the procedure in LS-281 (8), then compared to the recompacted BRD and MRD to determine the level of field compaction. This compaction data is presented in Table 6, where the experimental values represent an average of two test results: those from material located under the wheel tracks and between the wheel tracks.

In examining the voids portion of the Marshall testing of the northbound sample in Table 3a, one notices that all except the 2nd lift are below today's specification of 3%-5% voids. Even compared to the old specification of 2%-3% voids in place during construction, only the 3rd and 1st lifts were within this. This low void problem was apparent even during construction, since the mix checks indicate that the 7th and 6th lifts were below the specified 2% minimum. The mix checks show that this surface mix actually contained only 1.9% voids, while tests on the recovered material show only 1.4% voids today. Although the designed void content for lift 6 was not available, records indicate that there were only 1.9% voids at the time of construction, while present testing indicated only 1.0% voids today.

Even more interesting is the fact that the first HL6 mix design used on Contract 71-14 was chosen at 1.9% voids, which is under the 2% minimum allowable at that time. Although the design was for 1.9%, the mix checks indicate a rise in voids to 2.3% during placement. The void checks performed under this study are substantially less however, at 0.6% and 1.6% for the 4th and 5th lifts respectively. Lift 3 maintained this low trend with 2.1%, while lifts 2 and 1 were closer to today's accepted void content with 3.1% and 2.8% respectively.

Similar problems with voids exist in the southbound sample. The surface course of this sample was designed at 2.3% voids, although construction mix checks showed 1.7% and the present study showed only 0.3% voids. Our testing showed the 6th lift to be just above the mix design value of 2.0%; however mix checks indicated it only had 1.7% at the time of construction. Lift 5 is the only lift within the southbound sample, and only one of two in the entire study with present day test results showing greater than today's minimum void specification of 3.0%. This is ironic, because the mix design target value was under the 2.0% minimum specification at 1.8%. While mix design values and construction mix check results were not available for the bottom four lifts, present day tests show there are 2.7% voids in the 1st lift, but less than 2% in each of lifts 2, 3, and 4.

This problem with low voids does not appear to be reflected in the mix check VMA test results. In each case where this information was available, the mix check VMA was greater than the minimum specified by the mix design, and within the specification in place at that time.

Another point to note from Tables 3a and 3b are the high Marshall stability values for all the mixes in each sample. The results from the recompacted samples show stability values that are high, although oxidation of the AC and aging of the pavement likely contribute to this. Mix checks from both 1971 construction contracts indicate that the lifts placed at that time each exhibited stability values which were at least 20% greater than required by the mix design. With the exception of lift 5 southbound, the stability of each of these mixes during construction were greater, in fact, than required by today's specification.

Considering the recompacted densities shown in Table 6, one notices the field densities of the recovered samples are each at least 98.5% of the recompacted BRD, and in some cases exceed 100%. Also, with the exception of one lift, each has been compacted to at least 97% of the MRD.

(ii) Wheel Tracking Machine Test

A portion of the field material was tested using the Wheel Tracking Machine (WTM) utilizing the constant temperature test procedure described in Appendix B of (10). This WTM test was carried out on both undisturbed and recompacted materials.

The WTM test on undisturbed material was intended to examine relationships between adjacent lifts which may have an influence on rutting. Six specimens, each consisting of two consecutive lifts, were cut directly from the recovered blocks. These test specimens consisted of the 7th lift on the 6th lift, the 6th lift on the 5th lift, etc., down to the 2nd lift on the 1st lift. One sample of each combination was tested in the WTM apparatus. Each saw cut specimen was set into a WTM mould with a thin asphalt-screenings slurry applied to the sides of the mould to provide a bond between it and the test specimen.

The recompacted portion of the WTM testing involved laboratory manufactured specimens, each consisting of two lifts. A description of the sample preparation process is included in Appendix B of (10). A standard laboratory HL4 asphalt mix was used for the lower lift of each specimen, while the top lift consisted of recompacted material from each recovered block. One sample from each lift of each slab sample was tested in this manner. This method of WTM testing was used to determine the rutting susceptibility of each individual lift, without the influence of the lift(s) directly below.

It is recommended that more credibility be placed upon the recompacted WTM test than on the undisturbed sample results, since there is a greater chance for error inherent in the preparation of the undisturbed test specimens. For these samples, much care has to be given to ensure the saw cuts are absolutely straight and square. It is more difficult to obtain a proper fit between these specimens and the mould than for specimens manufactured in the laboratory. This increased difficulty translates into poorer repeatability for the WTM test on undisturbed specimens.

A comparison of WTM results on the northbound sample (Figure 6a) shows that there are two lifts, #5 and #4, exhibiting significantly more rutting than any others (summary below). After these, the greatest rutting appears in the 1st lift, with 60% less rutting than in lift 4.

Lift #	NB WTM rut depth (mm)	SB WTM rut depth (mm)
7 (surface)	3.0	4.7
6	3.2	3.6
5	6.7	2.0
4	5.9	2.8
3 (surface)	2.9	6.9
2	3.2	3.2
1	3.7	5.8

The poor rut results of both lifts 5 and lifts 4 were further substantiated by testing undisturbed samples using combinations of lifts 6 on 5 and lifts 5 on 4, with lifts 5 and 4 acting as binder courses for the respective overlying lifts. This resulted in approximately

22.1 mm of rutting for both tests, much higher than for WTM testing of other northbound undisturbed samples.

The recompacted specimens of the upper binder lifts from the southbound lane exhibited reasonably good resistance to WTM rutting, as shown by the above summary and the rutting comparison of Figure 6b. The problem lifts appear to be the traprock mixes of lifts 3 and 7, as well as lift 1 (HL6 binder). Of these, the 3rd lift exhibited 6.9 mm of rutting. This was the highest value of any lift tested in this investigation.

Looking at the undisturbed WTM testing of the southbound sample, it is clear that the combination of lift 2 on 1 exhibited more rutting than any other combination in this study with 22.9 mm. The degree of rutting decreases as the specimens progress closer to the surface, with the combination of lift 7 on 6 being the lowest at 4.2 mm.

Since the stresses in the lower lifts, i.e., lifts 1 to 3, of the pavement are greatly reduced relative to the surface layers, the poor WTM rut results of lifts 1 and 3 do not necessarily mean that these two lifts had contributed to the pavement rutting. However, the placement of the maintenance mix, lift 4, suggests that some type of problem was encountered with the original pavement structure of three lifts, prior to the resurfacing contract of 1971. This is substantiated by the WTM testing of this study, which indicates the instability of the lifts of that original pavement structure.

(iii) Gyratory Testing Machine

It has been shown that the Gyratory Testing Machine (GTM) can be a useful tool in evaluating the rut resistance of asphalt mixes (11). The GTM measures the shear strength of the asphalt mix, an important engineering property in rutting resistance. In light of this, part of the recovered material was tested by the GTM using the ASTM procedure described in (12).

It was recommended in (11) that GTM testing be conducted with a vertical pressure constant at 827 kPa (120 psi) while varying the number of revolutions between 30, 90, 300, and 500, as well as testing at 90 revolutions with 345 kPa (50 psi) and 552 kPa (80 psi) of vertical pressure respectively. One separate specimen of recovered material was tested at each of these settings. This procedure was followed for every lift of each slab, allowing for the determination of the bulk relative density (BRD), the gyratory stability index (GSI), gyratory shear factor (GSF), and gyratory shear strength (Sg), for each set of test parameters.

A widening of the graphical output of the GTM testing, the gyrograph plot, indicates a decrease in mix stability. Lifts 3, 4, 5, 6 and 7 northbound and lifts 3, 4 and 6 southbound were deemed to show such an instability. On the basis of the widest gyrograph plot representing the worst mix, the rankings of those lifts from best to worst after 500 revolutions at 827 kPa are: lifts 4, 3, 7, 5 and 6 for the northbound sample and 4, 3 and 6 for the southbound sample. These plots indicate that in both samples, the upper binder (lift 6) appears to show instability. This is critical at a depth where the pavement is subjected to maximum horizontal shear stresses under traffic loading.

To simplify reporting of the results, only the shear strength (Sg) results of the different layers are discussed. Unstable mixes with low Sg values of ≤100 kPa (14.5 psi) are found for the same northbound lifts as are mentioned above, but are ranked in slightly different order from high to low, i.e., 4, 7, 6, 3 and 5. Of the three southbound lifts mentioned above, only lift 6 had a low Sg. Lift 7 southbound produced a low response at 300 revolutions, but did not exhibit a widening gyrograph.

As shown in Figures 8a and 8b, all Sg values at 300 or 500 revolutions, representing minimal mix shear strengths available, fall well below the recommended GTM Sg of 276 kPa (40 psi) for rut resistant mixes (11). Those mixes that were badly rutted by the WTM (4th and 5th lifts northbound and 1st and 3rd lifts southbound) have Sg values ranging from 19 kPa to 257 kPa. The Sg results, especially for the top two layers, show that all of the mixes had very poor resistance to rutting.

GTM testing was also performed for some of the lifts on mixes reconstituted from extracted aggregates and new AC with the same penetration grade. This was to determine if aged in-situ mixes from a pavement would perform the same way as the newly reconstituted mixes produced by this technique. Mineral filler was used to replace pass 75 μm sieve material tainted or lost during the extraction process. The two mixes on which this experimentation was done, 5 northbound and 3 southbound, were chosen because of their poor performance in WTM testing. Figure 9a and 9b shows that the Sg values for the reconstituted mixes at different revolutions of compaction follow similar trends as the original GTM samples. The results for the HL1 mix are particularly close.

Although the AC grade for the reconstituted mixes is the same as that originally used, it was expected that because of lack of aging that the new AC would be softer and thereby produce lower Sg results. However, the mineral filler was finer graded (Figure 10) than the original pass 75 μm sieve material for which it was substituted and, as a result, acts as an AC extender producing stiffer mixes with higher Sg values.

5.0 DISCUSSION

The results of the percent crushed analysis indicated that the northbound binder lifts 4, 5, and 6 each contained more than the minimum 60% of one face crushed coarse aggregate. This meets the specification for binders of that time, and for normal binder mixes today, but does not meet today's specification for binders placed on high volume highways. Heavy Duty Binder (HDB) courses consisting of 100% crushed material were adopted by the MTO in the mid 1980's for use on all freeways and highways with greater than 5000 AADT per lane. Highway 400 falls into this category, and requires HDB for any construction today; in fact, HDB was used on Contract 90-48, the rehabilitation contract that repaired the rutted areas represented by these samples. In light of this, we can conclude that although the northbound binder courses put down in 1972 were adequate by the standards of that time, they have proven to be inadequate considering the loading to which they are subjected today. This inadequacy is reflected in the poor performance of lifts 4 and 5 in the WTM and of lift 5 in the GTM.

Comparatively, the three upper binders of the southbound location contain coarse aggregate that can be considered 100% crushed. Lifts 5 and 6 have 94% of the coarse aggregate with at least one face crushed, while lift 4 has 99% with one face crushed. In contrast to the upper binders of the northbound sample which had significantly less crushed material, the southbound binders exhibited better rut resistance as seen in the WTM results.

From the results of the WTM test on undisturbed specimens, it can be seen that the underlying lift can have significant effects on the overall rutting of all test specimens. For instance, the northbound undisturbed sample of lift 6 on lift 5 ruts badly; yet the recompacted sample with lift 6 material on a Durham mix base behaves very well. This is because the lift forming the base for the undisturbed specimen, i.e., lift 5, is a highly rut susceptible mix as shown by the test on top of a Durham mix base. Conversely, when lift 4 was on lift 3, substantially less rutting was observed than did with lift 5 over 4 or lift 6 over 5. This is because lift 3, a very stable traprock mix, provides a good stable base for

the lift 4 above, improving the rut resistance and reducing the rut depth of the undisturbed sample. To extend this theory further to the highway situation, a premium quality mix over a poor quality binder will still rut because of the weak support of the binder course.

It is interesting to note that, on the basis of recompacted WTM results, the amount of rutting is generally proportional to the sandstone and siltstone contents present in the three upper binders of the northbound sample. The carbonate/sandstone(and/or siltstone) splits are 49%/17%, 70%/9% and 73%/5% producing rut depths of 6.7, 5.9 and 3.2 mm for lifts 5, 4, and 6 respectively. In each of these three cases, the rut depths decrease as the difference between the amount of carbonate material and siltstone and/or sandstone material increases. The exception to this trend occurs in the 2nd lift which has 23% sandstone and siltstone but exhibits only 3.2 mm of rutting.

The majority of rutting in the southbound sample was caused by the failure of the HL1 surface course. Construction mix checks indicated that it was over-asphalted by 0.4%, had only 1.7% voids, and contained more than twice as much pass 75 mm material as specified by the mix design (although still within specification). The low void content was confirmed by this study's test results showing the mix contained only 0.3% voids.

The occurrence of longitudinal cracks on the right side of the southbound sample suggests that the lack of an adequate sub-base under the widening portion could have at least partially contributed to the rutting failure of the right side of this particular pavement section (see Figure 7). This subsidence type of failure mechanism acted in addition to the plastic flow failure of the HL1 surface course. As the documentation relating to the missing layers of the pavement could not be found, the question as to why those layers were not placed according to the contract drawings cannot be answered. However, this points to the importance of good field inspection and record keeping, so that deviations from the contract documentation are properly recorded for future reference.

It is difficult to prove that the wire mesh as shown in the drawings of Contract 71-14 had improved the performance of the northbound sample because of the thick pavement structure (≈450 mm). However, the northbound pavement did not show any tension cracking. One can only speculate that a similar reinforcing agent on the southbound joint widening might have improved its structural integrity, thereby reducing the failure as observed in the right wheel track of the southbound sample.

Since the placement of the hot mixes in 1971-72 under Contracts 71-12 and 71-14, they have been exposed to high summer temperatures of at least 30°C during the summer months of every year (Figure 2), particularly in the years of 1974 and 1988 when peak air temperatures greater than 35°C were recorded. Long periods of warm temperatures will increase the rut susceptibility of pavements, especially in conjunction with heavy traffic conditions.

6.0 CONCLUSIONS

As a result of this laboratory investigation it was concluded that :

i) The rutting failure of the northbound sample section was a result of the poor binder course lifts used immediately below the pavement surface. These mixes did not meet specifications, both past and present, in a number of areas listed below and the combined effect of these factors resulted in the severity of rutting:
   - low void contents in all lifts,
   - generally fine gradations with excessive amounts of 150 - 300 mm material in the binder course lifts, and
   - insufficient percentages of crushed coarse aggregates (75%) for freeway mixes compared to current requirements of 100%.

ii) The mix responsible for the rutting failure of the southbound sample section was the HL1 surface course which had a void content well below specification. Also, the rutting of the pavement at this particular location was aggravated by a lack of adequate thickness on the widening portion. The as-constructed pavement structure did not have the required pavement depth as specified in the contract drawings.

iii) In general, the mixes evaluated do not meet the more demanding requirements of current mix designs for highways with high traffic volume and heavy loading, as judged by their shear strengths obtained from the GTM results.

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TABLE 1 : Summer Temperature Intensities 1989-90

Location	Month	number of days with a peak temperature >= 30°C	number of days with a peak temperature >= 25°C	no. of times with 3 consecutive days of peak temp. >= 30°C	no. of times with 3 consecutive days of peak temp. >= 25°C
Northbound Sample (Woodbridge Climatological Station)	June 1989	4	19	1	5
	July 1989	12	29	2	9
	August 1989	5	21	1	6
	Summer total	21	69	4	20
	June 1990	3	21	0	4
	July 1990	9	23	2	7
	August 1990	7	23	1	6
	Summer total	19	67	3	17
Southbound Sample (Bradford Muck Research Climatological Station)	June 1989	1	14	0	3
	July 1989	10	26	2	8
	August 1989	2	14	0	3
	Summer total	13	54	2	14
	June 1990	2	17	0	4
	July 1990	6	23	1	6
	August 1990	3	22	0	6
	Summer total	11	62	1	16

TABLE 2a : Mix Design History, Northbound Sample

Materials and Properties	Cont. 58-291 HL6 Binder	Cont. 58-291 HL1 Surface	Cont. 71-14 HL6 Binder Mix Design 1 [1]	Cont. 71-14 HL6 Binder Mix Design 2 [2]	Cont. 71-14 HL1 Surface
Applies to Lift #	1 , 2	3	4 , 5	6	7
C.A. Source	Stouffville	Ontario Rock, Havelock	Caledon Sa.&Gr., Caledon	Caledon Sa.&Gr., Caledon	Armstrong, Marmora
Screenings Source	none	Armstrong, Georgetown	1 Dufferin, Milton	2 Dufferin, Milton	1 Dufferin, Milton
Sand Source	Armstrong, Caledon	Armstrong, Caledon	1 Crawford, Maple	3 Superior, Maple	3 Harris, Goodwood
Retained #4 (%)	na	52	45	50	45
Pass #4 (%)	na	48	55	50	55
AC Content (%)	4.4	5.4	5.5	5.0	5.5
AC Source, Type	B.A., 85/100	B.A., 85/100	Gulf, 85/100	Gulf, 85/100	Gulf, 85/100
Voids (%)	na	2.0	1.9	na	2.5
Flow (0.25 mm)	na	10.0	13.9	na	11.4
Stability (N)	na	7560	7250	na	8180
VMA (%)	na	16.0	15.3	na	15.7

na = not available
1 = used prior to August 27, 1971
2 = used after August 27, 1971
B.A. = British American

TABLE 2b : Mix Design History, Southbound Sample

Materials and Properties	Cont. 51-33 HL6 Binder	Cont. 51-33 HL1 Surface	Cont. 71-12 HL6 Binder Mix Design 1 [1]	Cont 71-12 HL6 Binder Mix Design 2 [2]	Cont. 71-12 HL1 Surface
Applies to Lift #	1 , 2	3	5	6	7
C.A. Source	Uhthoff	"Trap"	Limestone Prod., Uhthoff	Limestone Prod., Uhthoff	Armstrong, Marmora
Screenings Source	Western	King	1 King Scr. Sand, Tottenham	1 Limestone Prod., Uhthoff	1Limestone Prod., Uhthoff
Sand source	McVety	Andrews	1 King Bl. Sa. Tottenham	4 King Bl. Sa.#3 Tottenham	4 King Bl. Sa.#3 Tottenham
Retained #4 (%)	na	na	45	45	45
Pass #4 (%)	na	na	55	55	55
AC Content (%)	na	na	5.4	5.2	5.1 [3]
AC Source, Type	B.A., 85/100	B.A., 85/100	I.O.L., 85/100	I.O.L., 85/100	Shell, 85/100
Voids (%)	na	na	1.8	2.0	2.3
Flow (0.25 mm)	na	na	13.5	11.0	9.5
Stability (N)	na	na	6000	7250	7650
VMA (%)	na	na	14.7	14.5	15.7

1 = used prior to September 7, 1971, normal mix
2 = used after September 7, 1971
3 = comment on 71-12 Bituminous Construction Report : "AC cut to 5.0 % to increase voids"

na = not available
B.A. = British American
I.O.L. = Imperial Oil Limited

TABLE 3a : Marshall Properties and Mix Proportions, Northbound Sample

Lift, Mix Type	Comparison	Stability (N)	Flow (0.25 mm)	VMA (%)	Voids (%)	AC (%)	% Pass 4.75 mm	% Pass 75 µm
7th, HL1	Mix Design	8180	11.4	15.7	2.5	5.5	55	3.2
	Mix Checks	10590	12.2	16.6	1.9	5.6	58	6.4
	Recompacted Mix	17960	16.0	- -	1.4	4.9	51.6	5.5
6th, HL6	Mix Design	na	na	na	na	5.0	50	3.0
	Mix Checks	11230	14.1	14.4	1.9	5.4	53.3	8.4
	Recompacted Mix	13230	17.6	- -	1.0	5.1	53.2	6.7
5th, HL6	Mix Design	7250	13.9	15.3	1.9	5.5	55	4.4
	Mix Checks	11360	12.9	15.8	2.3	5.5	50.7	8.6
	Recompacted Mix	14710	16.2	- -	1.6	5.6	62.3	7.1
4th, HL6	Mix Design	7250	13.9	15.3	1.9	5.5	55	4.4
	Mix Checks	11360	12.9	15.8	2.3	5.5	50.7	8.6
	Recompacted Mix	13740	13.5	- -	0.6	5.3	56.2	5.9
3rd, HL1	Mix Design	7560	10.0	16.0	2.0	5.4	48	2.4
	Mix Checks	8380	12.1	16.1	2.6	5.4	51.3	na
	Recompacted Mix	15140	15.0	- -	2.1	5.1	47.8	6.2
2nd, HL6	Mix Design	na	na	na	na	4.4	na	na
	Mix Checks	na	na	na	na	4.5	42.3	na
	Recompacted Mix	18820	11.2	- -	3.1	3.9	41.3	4.2
1st, HL6	Mix Design	na	na	na	na	4.4	na	na
	Mix Checks	na	na	na	na	4.5	42.3	na
	Recompacted Mix	17640	15.6	- -	2.8	4.2	41.0	5.0

Note : Recompacted Mix VMA not calculated since specific gravities of individual aggregates were unavailable.
Marshall mix designs and mix check values unavailable for lifts 6, 2, and 1.
Mix design gradations unavailable for lifts 2 and 1.
Mix check pass 75 µm unavailable for lifts 3, 2, and 1.

TABLE 3b : Marshall Properties and Mix Proportions, Southbound Sample

Lift, Mix Type	Comparison	Stability (N)	Flow (0.25 mm)	VMA (%)	Voids (%)	AC (%)	% Pass 4.75 mm	% Pass 75 µm
7th, HL1	Mix Design	7650	9.5	15.7	2.3	5.1	55	2.0
	Mix Checks	9680	11.6	16.0	1.7	5.5	56.1	4.7
	Recompacted Mix	15410	15.8	- -	0.3	4.8	50.9	4.8
6th, HL6	Mix Design	7250	11.0	14.5	2.0	5.2	55	2.0
	Mix Checks	10860	12.0	15.7	1.7	5.7	58.6	na
	Recompacted Mix	15110	13.2	- -	2.2	5.1	59.3	4.3
5th, HL6	Mix Design	6000	13.5	14.7	1.8	5.4	55	2.6
	Mix Checks	8500	9.9	16.6	2.5	5.8	57.0	na
	Recompacted Mix	16020	10.8	- -	3.1	5.1	58.7	4.9
4th, HL6	Recompacted Mix	17400	13.0	- -	1.6	6.0	65.4	6.5
3rd, HL1	Recompacted Mix	14530	7.0	- -	1.4	4.6	50.7	6.0
2nd, HL6	Recompacted Mix	14550	10.2	- -	1.8	4.7	40.1	2.8
1st, HL6	Recompacted Mix	14640	8.7	- -	2.7	4.8	54.0	4.8

Note : Recompacted Mix VMA not calculated since specific gravities of individual aggregates were unavailable

Mix designs and mix checks unavailable for lifts 4, 3, 2, and 1

TABLE 4 : Recovered AC Properties

Slab	Lift	Kin. Visc. (mm^2/s) @ 135°C	Abs. Visc. (Pa · s) @ 60°C	Penetration (0.1 mm) @ 10°C	@ 25°C	@ 40°C
NB	7	486	362	17	50	227
	6	605	560	14	44	198
	5	618	569	12	36	165
	4	562	485	15	41	198
	3	703	1773	12	36	111
	2	730	1328	15	40	131
	1	569	672	15	45	161
SB	7	572	566	12	45	190
	6	616	552	12	44	194
	5	495	436	12	49	227
	4	575	712	15	50	165
	3	303	159	13	55	275
	2	295	207	12	51	252
	1	360	327	10	40	198

TABLE 5 : Coarse Aggregate Description and Percent Crushed

Slab	Lift, Mix Type	Description	% With One or More Faces Crushed	% With Two or More Faces Crushed	% WithThree or More Faces Crushed	% With All Faces Crushed	% Uncrushed
NB	7th, HL1	42% trap, 32% granite, 24% volcanic	processed material, assumed 100% crushed				
	6th, HL6	73% carbonate, 15% granite, 5% siltstone	75	62	11	29	25
	5th, HL6	49% carbonate, 17% granite, 14% gneiss 12% sandstone, 5% siltstone	73	61	14	28	27
	4th, HL6	70% carbonate, 10% greywacke 9% sandstone, 5% chert	74	63	18	23	26
	3rd, HL1	100% trap	processed material, assumed 100% crushed				
	2nd, HL6	59% carbonate, 13% sandstone 13% granite, 10% siltstone	59				
	1st, HL6	86% carbonate, 6% granite	53				
SB	7th, HL1	51% trap, 34% volcanic, 15% granite	processed material, assumed 100% crushed				
	6th, HL6	100% carbonate	92	92	92	92	8
	5th, HL6	100% carbonate	95	93	93	92	5
	4th, HL6	100% carbonate	99	99	99	99	1
	3rd, HL1	81% trap, 15% granite,	processed material, assumed 100% crushed				
	2nd, HL6	93% carbonate, 6% gneiss	92				
	1st, HL6	78% carbonate, 12% gneiss, 9% granite	70				

TABLE 6 : Recompacted Densities

Slab	Lift, Mix Type	BRD	MRD	Field Density	Field Density as % of BRD	Field Density as % of MRD
NB	7th, HL1	2.661	2.692	2.654	99.7	98.6
	6th, HL6	2.477	2.490	2.465	99.6	99.0
	5th, HL6	2.447	2.475	2.436	99.6	98.4
	4th, HL6	2.446	2.468	2.458	100.5	99.4
	3rd, HL1	2.606	2.628	2.574	98.8	98.0
	2nd, HL6	2.433	2.505	2.428	99.9	97.0
	1st, HL6	2.448	2.482	2.412	98.5	97.2
SB	7th, HL1	2.692	2.698	2.695	100.2	99.8
	6th, HL6	2.467	2.486	2.432	98.6	97.8
	5th, HL6	2.443	2.489	2.413	98.8	97.0
	4th, HL6	2.442	2.468	2.430	99.5	98.5
	3rd, HL1	2.599	2.616	2.581	99.3	98.7
	2nd, HL6	2.457	2.488	2.444	99.5	98.3
	1st, HL6	2.427	2.486	2.391	98.5	96.2

FIGURE 1 : Highway 400 Traffic Growth, 1955-1988

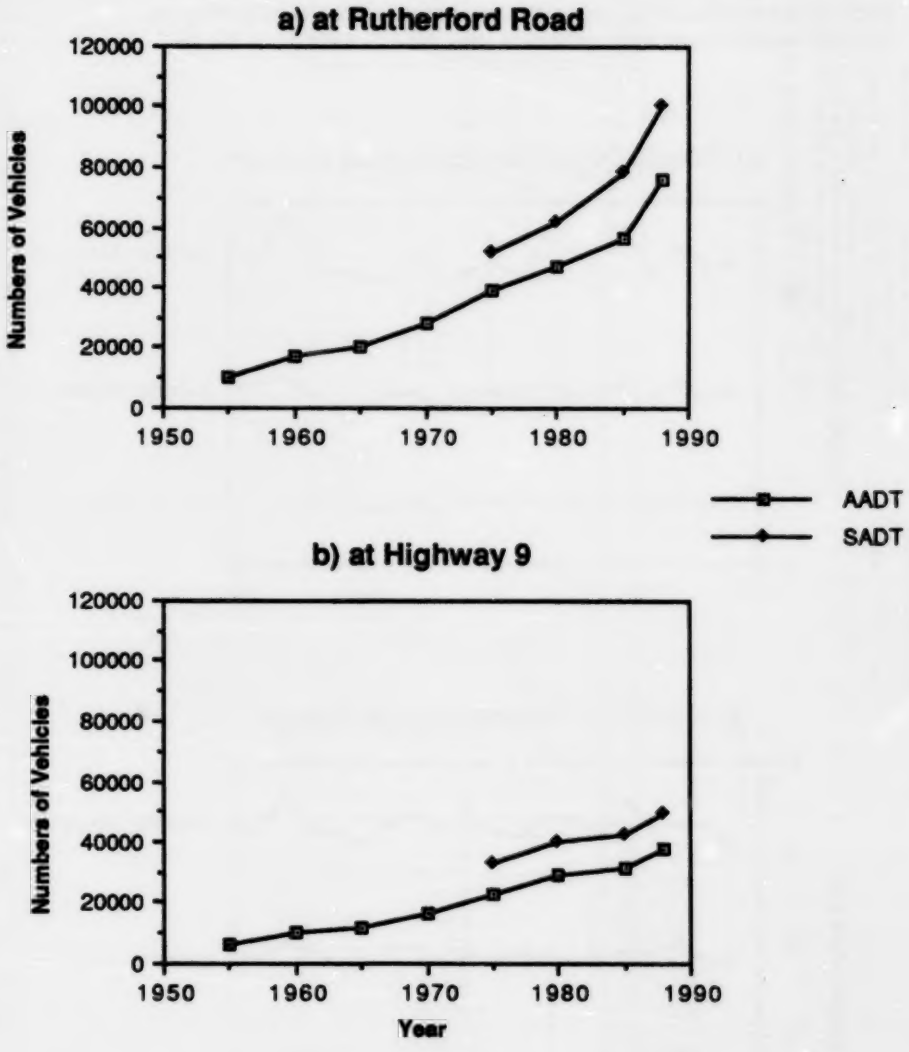


FIGURE 2 : Summer Temperature Records

Summer Maximum = highest daily temperature reading during June, July, August
Summer Mean = average of monthly mean temperatures of June, July, August
Annual Mean = average of the 12 monthly mean temperatures

(Woodbridge Climatological Station data used for Fig 2a,
Bradford Springdale Climatological Station data used for 1971-73 of Fig 2b,
Bradford Muck Research Climatological data used for 1974-90 of Fig 2b)

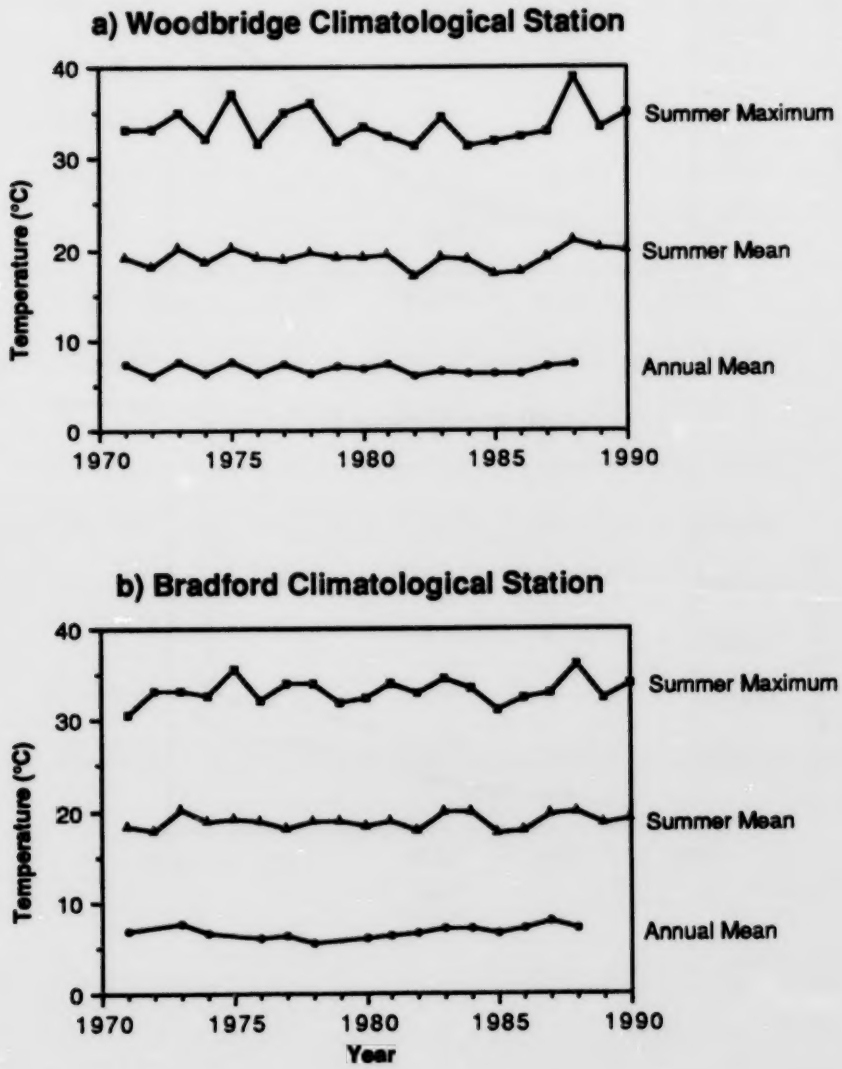
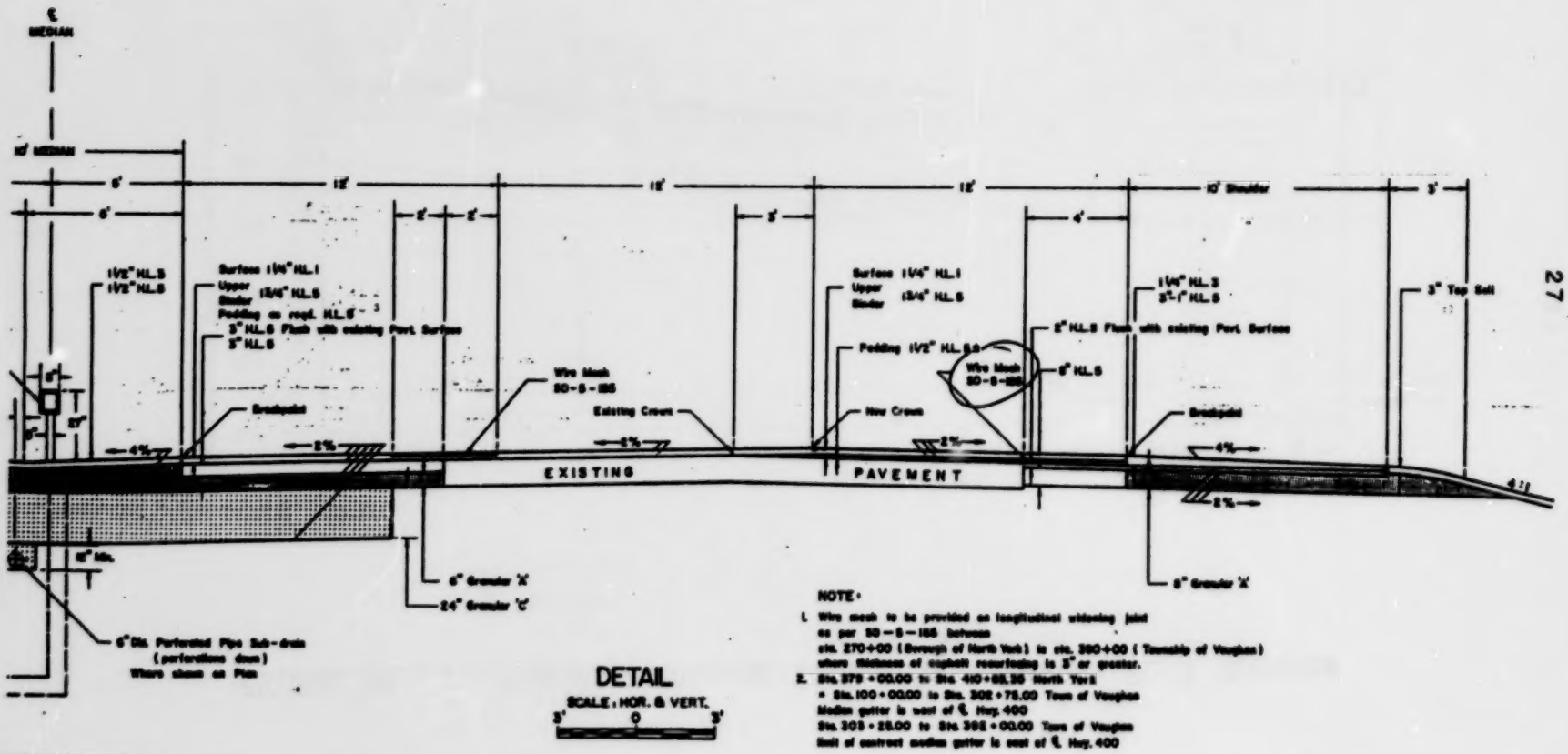


FIGURE 3a : Typical Northbound Section from Contract 71-14 Drawings



DETAIL
SCALE: HOR. & VERT.
3' 0 3'

NOTE:

1. Wire mesh to be provided on longitudinal widening joint as per 50-8-155 between sta. 270+00 (Borough of North York) to sta. 280+00 (Township of Vaughan) where thickness of asphalt resurfacing is 3" or greater.
2. Sta. 279+00.00 to Sta. 410+65.35 North York
   + Sta. 100+00.00 to Sta. 302+75.00 Town of Vaughan
   Median gutter is west of ℄ Hwy. 400
   Sta. 303+25.00 to Sta. 392+00.00 Town of Vaughan
   limit of contract median gutter is east of ℄ Hwy. 400

FIGURE 3b : Typical Southbound Section from Contract 71-12 Drawings

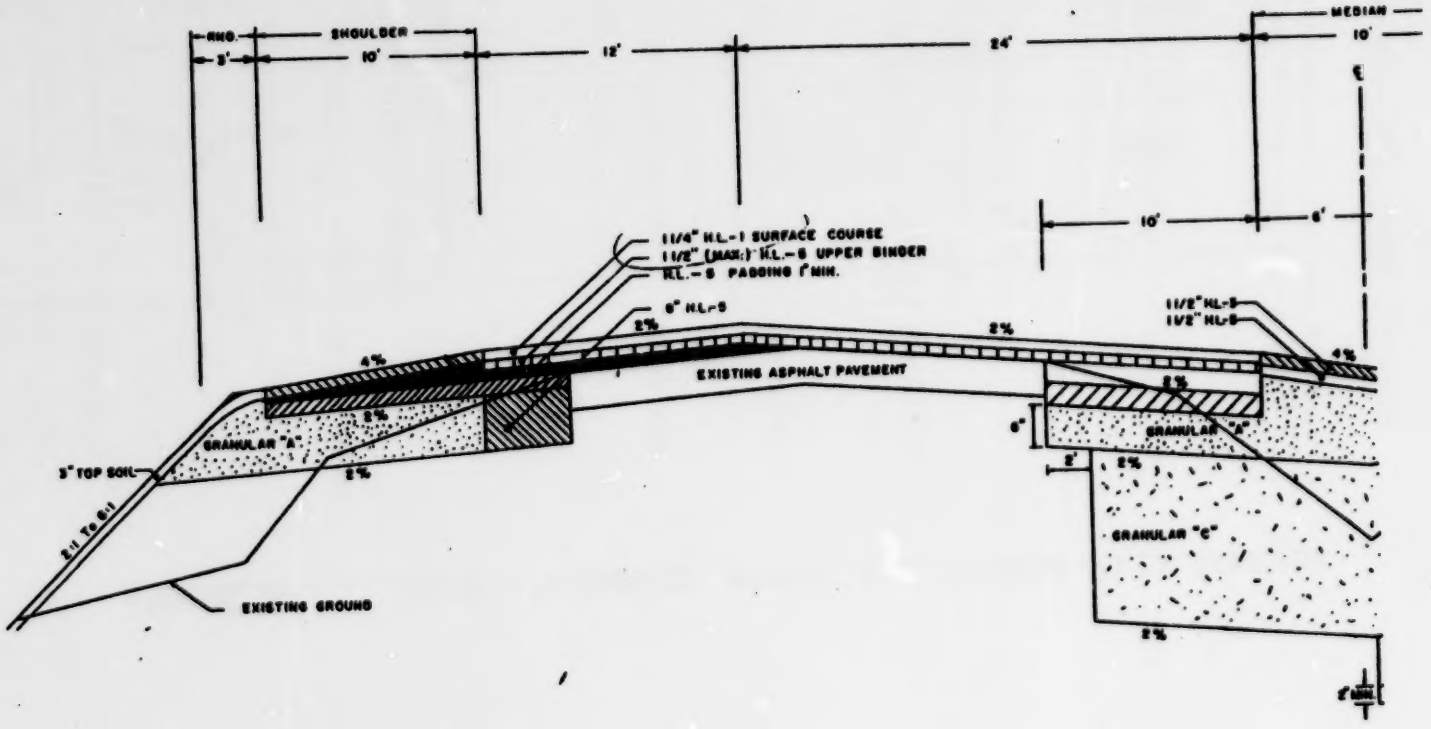
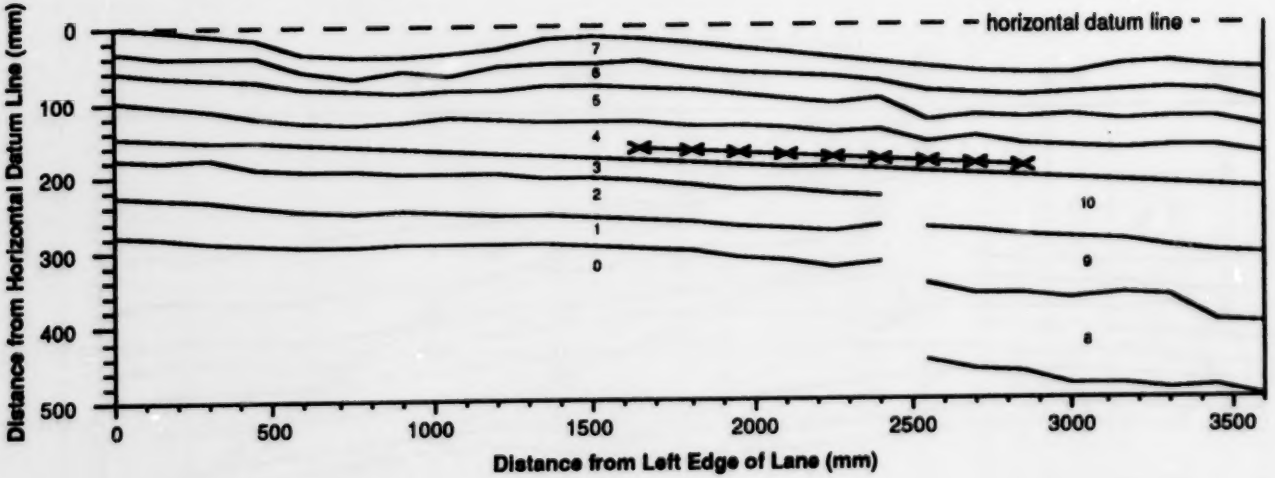


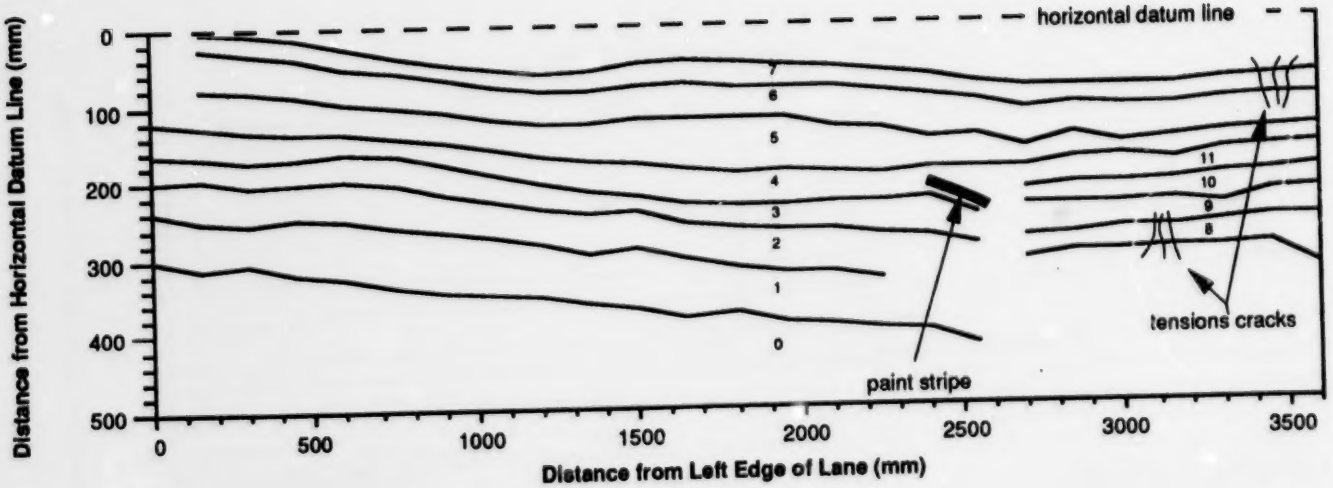
FIGURE 4a : Northbound Rutting Profile



7	top lift (HL1)
6	6th binder
5	5th binder
4	4th binder
3	3rd binder (HL1)
2	2nd binder
1	1st binder
0	granular subbase

⋈	steel mesh
10	3rd extension binder
9	2nd extension binder
8	1st extension binder

FIGURE 4b : Southbound Rutting Profile



7	top lift (HL1)
6	6th binder
5	5th binder
4	4th binder
3	3rd binder (HL1)
2	2nd binder
1	1st binder
0	granular subbase
11	4th extension binder
10	3rd extension binder
9	2nd extension binder
8	1st extension binder

FIGURE 5a : Semi-Log Grading Chart for HL6 Northbound

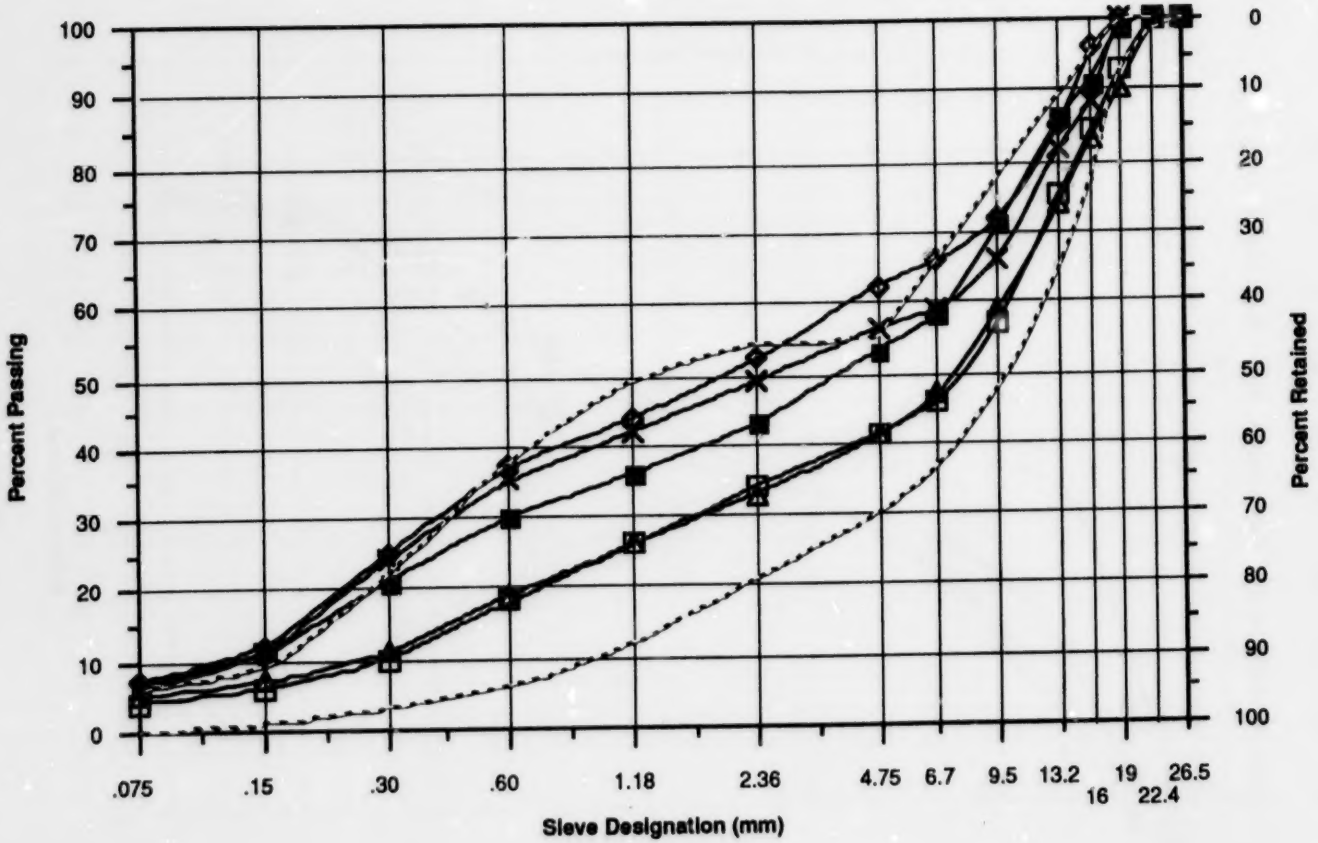


FIGURE 5b : Semi-Log Grading Chart for HL6 Southbound

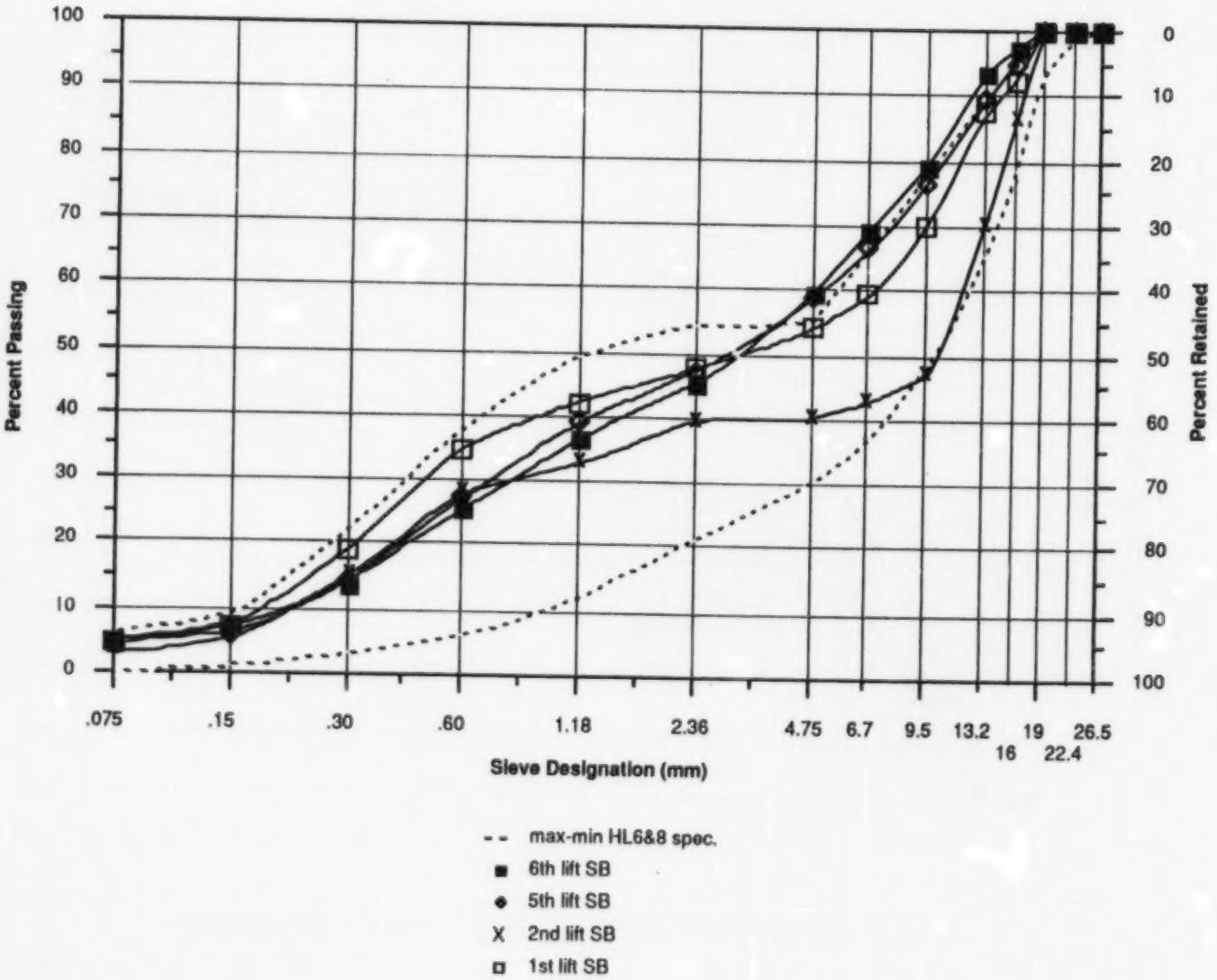


FIGURE 5c : Semi-Log Grading Chart for HL1 and HL3

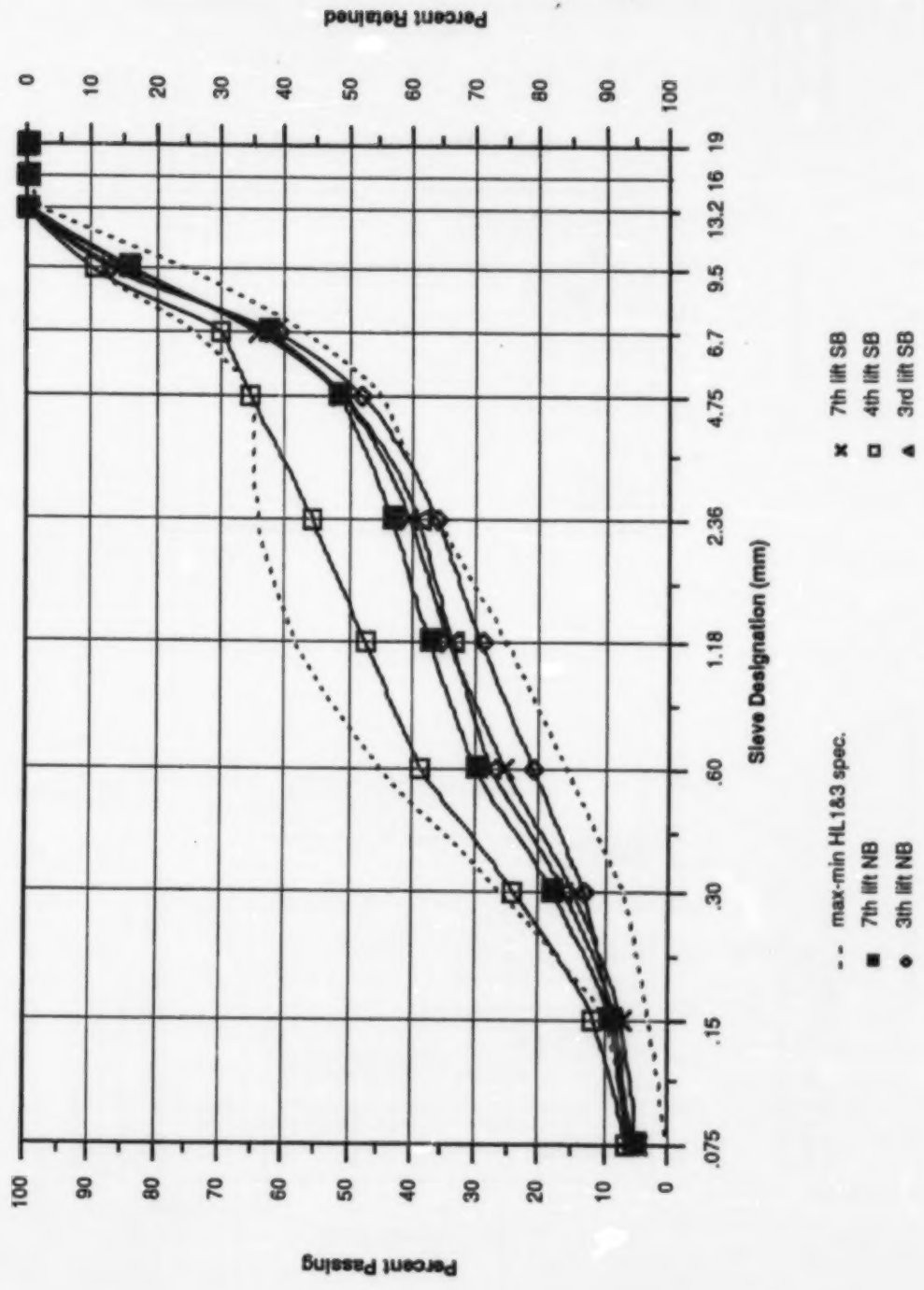


FIGURE 6 : Rut Depth of Each Lift

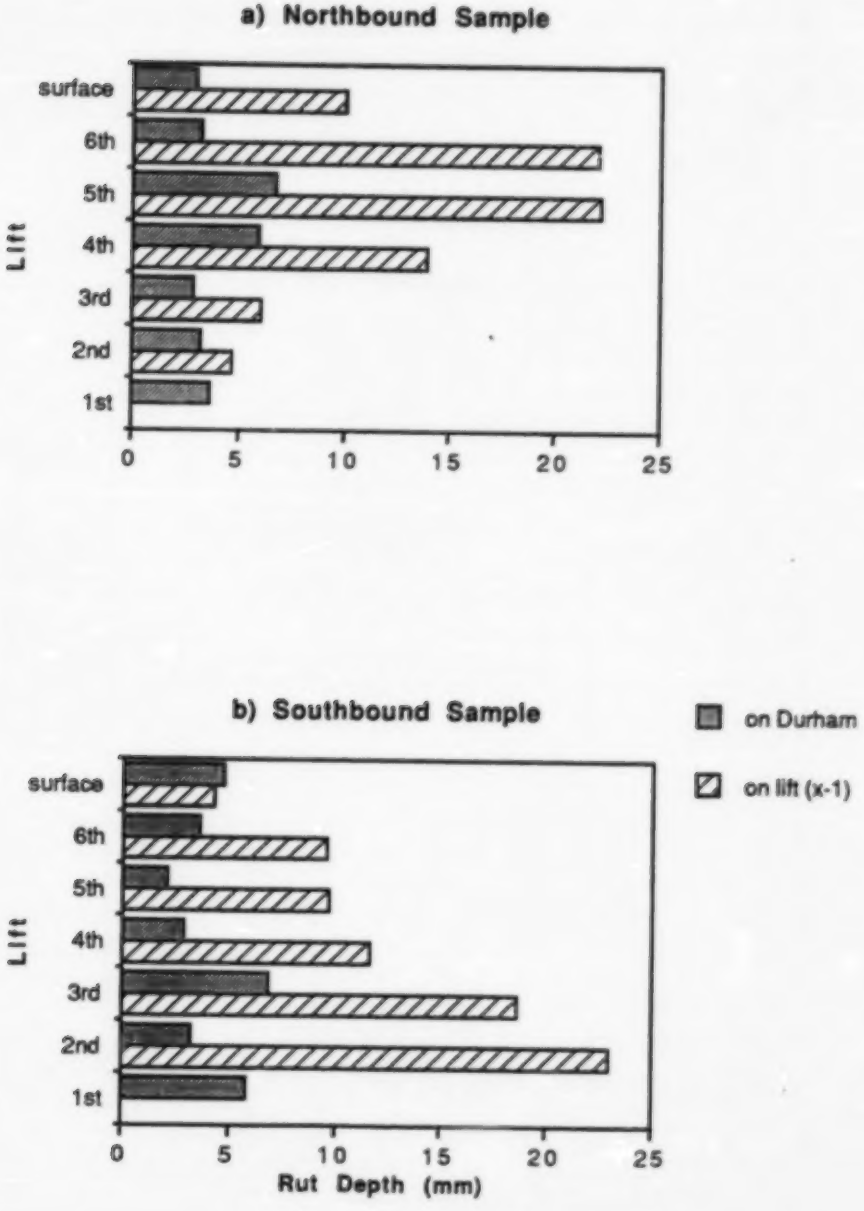


FIGURE 7 : Possible Failure Mechanism of the Right Side of the Southbound Sample

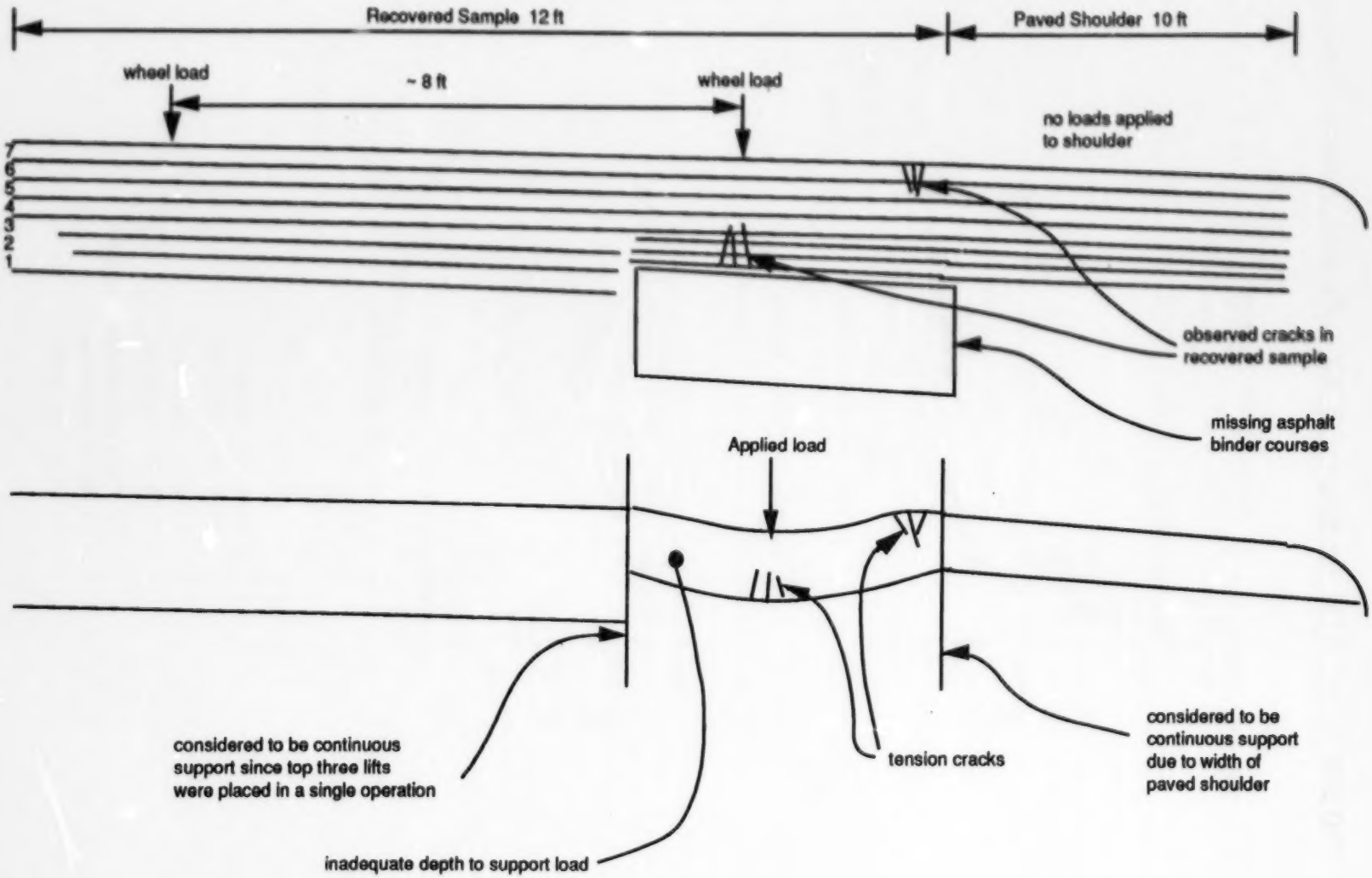


FIGURE 8 : GTM Sg Values @ 827 kPa and Various Revolutions

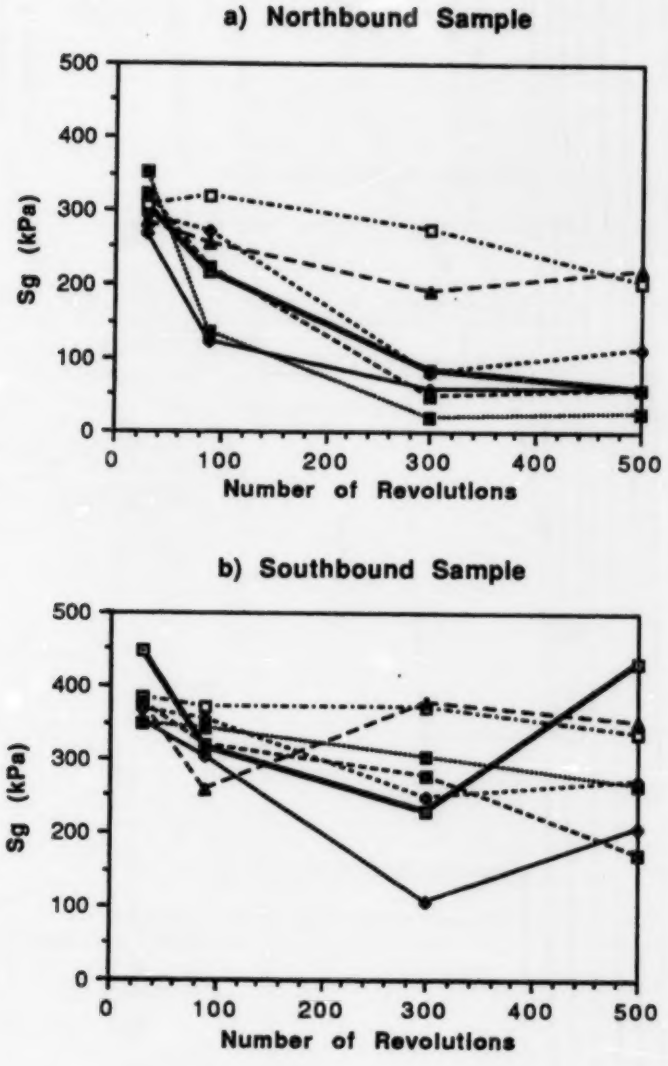


FIGURE 9 : GTM Sg Comparison of In-Situ and Reconstituted Mix @ 827 kPa and Various Revolutions

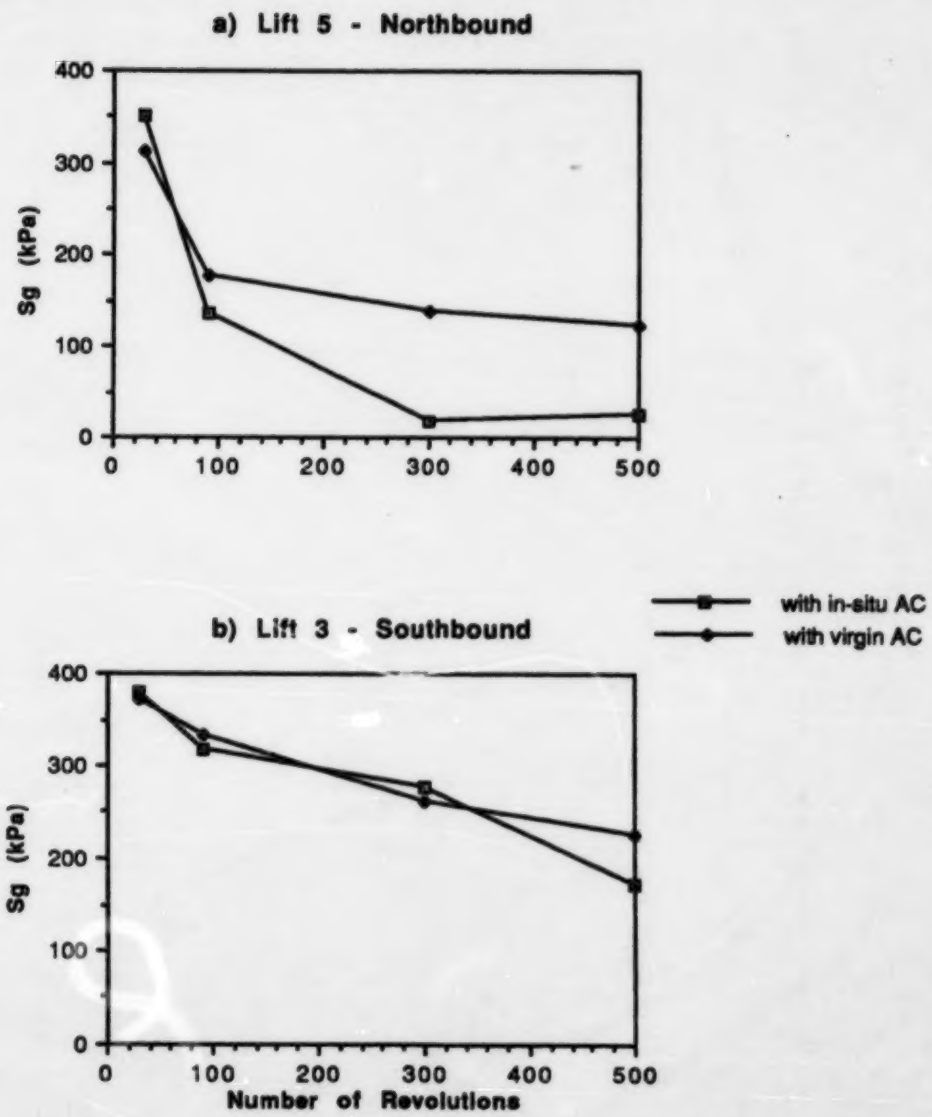


FIGURE 10 : Gradations of Mineral Filler and Pass 75 µm Reconstituted Mix